CHAPTER 30

ROADWAY

30.1 General

All design criteria in this manual are minimums, and these design criteria do not eliminate the responsibility of the designer to exceed these minimum standards where good engineering practice dictates. All materials and workmanship shall conform to these Standards and to the District of Columbia, current **Standard Specifications for Highway and Structures.**

30.2 AASHTO Policy

AASHTO, A current Policy on Geometric Design of Highways and Streets, the **Highway Capacity Manual,** was used as a reference within this chapter.

30.3 ADA Requirements

All designs for roadways shall conform to **ADA** requirements.

30.4 Functional Classification

30.4.1 General

Highway classification refers to a process by which roadways are classified into a set of sub-systems based on the way each roadway is used. Central to this process is an understanding that travel rarely involves movement along a single roadway. Rather, each trip or sub-trip initiates at a land use, proceeds through a sequence of streets, roads and highways, and terminates at a second land use.

The highway classification process is required by federal law. Each state must assign roadways into different classes in accordance with standards and procedures established by the Federal Highway Administration.

30.4.2 Functional Highway Systems in Urbanized Areas

DDOT has adopted a ***Functional Street Classification Plan*** based on traffic volumes, land use, and expected growth. The four functional highway systems identified are:

- Urban principal arterials (streets)
- Minor arterials (streets)
- Collectors (streets)

- Local streets

Each classification has design criteria that maintains and protects the primary purpose of the roadway.

30.5 Design Controls

30.5.1 General

The location and geometric design of highways is affected by numerous factors and controlling features; these may be considered in two broad categories. They are, Primary Controls (Highway Classification, Topography and Physical Features and Traffic) and Secondary Controls (Design Speed, Design Vehicle and Capacity).

30.5.2 Traffic Volume

For planning and design purposes, the demand of traffic is generally expressed in terms of the design-hourly volume (DHV), predicated on the design year. The design year for new construction and reconstruction is to be 20 years beyond the anticipated date of construction and 10 years beyond the anticipated date of construction for resurfacing, restoration or rehabilitation projects and Plans, Specifications, and Estimate (PS&E) are complete for bids.

30.5.3 Design Speed

Except for local streets where speed controls are frequently included intentionally, every effort should be made for a practical design speed to attain a desired degree of safety, mobility, and efficiency. The existing streets will be upgraded for a minimum speed as listed in the table 30-A. Once selected, all pertinent features of the highway should be in correlation to obtain a balanced design.

The design speed (Mph) as it relates to the posted speed (Mph) is shown in Table 30-A (see next page).

Table 30-A:
Design Speed vs. Posted Speed

Posted Speed	Design Speed Existing Highway	Proposed Design Speed Reconstruction Streets, Highways, or Alignments
20 Mph	25 Mph	30 Mph
25 Mph	30 Mph	35 Mph
30 Mph	35 Mph	40 Mph
35 Mph	40 Mph	45 Mph
40 Mph	45 Mph	50 Mph
45 Mph	50 Mph	55 Mph
50 Mph	55 Mph	60 Mph
55 Mph	60 Mph	65 Mph

30.5.4 Desired Design Speed for New Streets

New streets/highways shall be designed for a minimum speed as listed in Table 30-B:

Table 30-B

DESCRIPTION	DESIGN SPEED
Local	35
Collector	35
Minor arterials	40
Urban principal arterials	45
Expressways	65

Generally, for freeways and the Interstate system, the design speed shall be 70 Mph. When it is not practical to attain the desired speed in urban areas, the Interstate highway or freeway design speed shall not be less than 55 Mph.

Design speeds shall be selected in minimum increments of 5 Mph.
While it may be necessary to vary the design speed along certain highway sections for economic reasons, a uniform design speed should be maintained. Where a change in design speed is necessary, the maximum change should not exceed 10 Mph.

For roadways where it is not possible or feasible to maintain a 10 mph design speed over the posted speed limit, the designer shall maintain the greatest design speed possible but in no case shall it be less than the posted speed limit.

30.5.5 Highway Capacity

To determine the capacity for a particular highway design, the designer shall refer to the most recent edition of the Highway Capacity Manual (**HCM)** for guidance.

30.6 Basic Geometric Design Elements

30.6.1 General

Geometric highway design pertains to the visible features of the highway. It may be considered as the tailoring of the highway to the terrain, to the controls of the land usage, and to the type of traffic anticipated.

This section covers design criteria and guidelines on the geometric design elements that must be considered in the location and the design of the various types of highways (Refer to Table 30-C). Included are criteria and guidelines on sight distances, horizontal and vertical alignment, and other features common to the several types of roadways and highways.

In applying these criteria and guidelines, it is important to follow the basic principle that consistency in design standards is of major importance on any section of road. The highway should offer no surprises to the driver in terms of geometrics. Problem locations are generally where minimum design standards are introduced on a section of highway where otherwise higher standards should have been applied. The ideal highway design is one with uniformly high standards applied consistently along a section of highway, particularly on major highways designed to serve large volumes of traffic at high operating speeds.

The geometry of the new roadways must allow for the easy operation, maneuvering, turning, parking, standing, or emergency stopping of all types of running vehicles, including emergency vehicles. Determine if bus and/or truck operations are involved and if the buses can make the necessary turns.

For additional information and criteria relative to geometric design elements, refer to the **AASHTO Green Book**.

Table 30-C:
Geometric Design Elements

Geometric Design Elements	New or Reconstruction Projects
Typical Lane Width	10’ minimum
Typical usable shoulder width or parking lane	8 ft. to 10 ft.
Typical cross slope for driving lanes	1.0 to 4 percent*
Maximum degree of horizontal curve	5 degrees
Maximum superelevation	6 percent
Horizontal clearance to obstructions normally provided	2.0’
Maximum percent grade	8 percent (new development)
Minimum stopping sight distance	300’
Minimum roadway width on structures less than 200 ft. long	24’
Typical structural capacity	HS-25

NOTE: Minimum clearance over roadways in the District is 14’ – 6”. Minimum vertical clearances of roadways under structures are given in the **Structures** chapter within this manual.

* The parking lane, which may be used as a through lane at times, may have a cross slope of 4.0 percent in order to meet grades and elevations and on Local streets the parking lane may have a maximum

30.6.2 Sight Distances

Sight distance represents the continuous length ahead, along a roadway, that an object of specified height is continuously visible to the driver. For the safe and efficient operation of a vehicle on a highway, proper sight distance should be provided to enable drivers traveling at or near the design speed to control the operation of their vehicles to avoid striking an unexpected object or to stop before reaching a stationary object in their path.

The criteria for measuring sight distance are dependent on the height of the driver's eye above the pavement surface, the specified object height above the pavement surface, and the height of sight obstructions within the line of sight. For calculating both stopping and passing sight distances, the height of the driver's eye above the pavement surface shall be considered as 3.5 ft. For stopping sight distance calculations, the height of object shall be considered as 6 in. above the pavement surface. For passing sight distance calculations, the height of object shall be considered as 3.5 ft. above the pavement surface.

On tangents, the obstruction that limits the driver's sight distance is the road surface at some point on a crest vertical curve. On horizontal curves, the obstruction that limits the driver's sight distance may be the road surface at some point on a crest vertical curve. It may be some physical feature outside of the traveled way, such as a longitudinal barrier, a bridge-approach fill slope, a tree, foliage, or the back-slope of a cut section. Accordingly, all highway construction plans should be checked in both the vertical and horizontal plane for sight distance obstructions.

Table 30-D shows the standards for passing and stopping sight distance related to design speed.

Table 30-D:
Sight Distance for Design

	SIGHT DISTANCE IN FT.		
Design Speed (Mph)	Stopping Desirable	Stopping Minimum	Passing Minimum*
25	150	150	---
30	200	200	1100
35	250	225	1315
40	325	275	1500
45	400	325	1650
50	475	400	1800
55	550	450	1950
60	650	525	2100
70	850	625	2500

*Not applicable to multi-lane highways

The passing sight distance for upgrades should be greater than minimum. To enhance safety on new construction projects where the design speed is 70 Mph, it is recommended that a minimum stopping sight distance of 725 ft. be used, which provides for an average running speed of 65 Mph. The stopping sight distances shown in Table 30-D should be increased when sustained downgrades are steeper than 3 percent. Increases in the stopping sight distances on downgrades are indicated in the **AASHTO Green Book**.

Sight distance is the distance necessary for a vehicle operator to perform expected functions and be able to do so without causing a hazard for the driver or other vehicle operator for the specific design speed of the street. Vehicles shall perform moves without causing other vehicles to slow from the average running speed. In no case shall the distance be less than the stopping sight distance. This includes visibility at intersections and driveways as well as around curves and roadside encroachments. Stopping sight distance is calculated according to the **AASHTO Green Book**. Object height is 6 in. above road surface and viewer's height is 3.50 ft. above road surface.

Where an object off the pavement restricts sight distance, the minimum radius of curvature is determined by the stopping sight distance exists at all property lines except in the sight-distance easements that may be required to preserve the needed sight distance.

Stopping sight distance on horizontal curves is based upon lateral clearance from the inner edge of pavement to sight obstruction, for various radii of inner edge of pavement and design speeds. The position of the driver's eye and the object sighted shall be assumed to be 6 ft. from the inner edge of pavement, with the sight distance being measured along this arc. Stopping sight distances are given in Table 30-E.

Table 30-E:
Stopping and Passing Sight Distance

DESIGN SPEED (MPH)	STOPPING SIGHT DISTANCE (FT.)	PASSING SIGHT DISTANCE (FT.)
20	125	800
25	150	1000
30	200	1100
35	250	1300
40	275	1500
45	325	1650
50	400	1800

NOTE: From **AASHTO Green Book** Table III-1, Table III-5 and Table VII-3

On Arterials and Collectors, the corner sight distance shall provide for vehicles to enter traffic and accelerate to the average running speed. All sight-distance triangles must be shown on the street plan/profile plans. All sight distances must be within the public ROW or a sight distance easement. If the line of sight crosses onto private property, a "Sight Distance Easement" shall be indicated on the plat to meet the required sight distance. The District shall obtain from the property owner the required easement or ROW to be dedicated to the District. In any event, the District shall try and work with the property owner to establish an unobstructed sight distance triangle.

Any object within the sight triangle more than 30 in. above the flow-line elevation of the adjacent street shall constitute a sight obstruction, and shall be removed or lowered. Such objects include, but are not limited to, berms, buildings, and parked vehicles parked on private property, cut slopes, hedges, trees, bushes, utility cabinets or tall crops. Since parked vehicles are under the control of the District, parked vehicles shall not be considered an obstruction for design purposes. The city may limit parking to protect visibility. The sight distance shall be measured to the centerline of the closest through-lane in both directions.
In no case shall any permanent object encroach into the line-of-sight of any part of the sight-distance triangle. Street trees required by the District are an exception to this requirement. Trees are permitted if pruned up to 8 ft.

30.7 Horizontal Alignment

30.7.1 General

In the design of horizontal curves, it is necessary to establish the proper relationship between design speed, curvature, and superelevation. Horizontal alignment must afford at least the minimum stopping sight distance for the design speed at all points on the roadway.

The major considerations in horizontal alignment design are: safety, grade, and type of facility, design speed, topography, and construction cost. In design, safety is always considered, either directly or indirectly. Topography largely controls both curve radius and design speed. The design speed, in turn, controls sight distance, but sight distance must be considered concurrently with topography because it often demands a larger radius than the design speed. All these factors must be balanced to produce an alignment that is safe, economical, in harmony with the natural contour of the land and, at the same time, adequate for the design classification of the roadway or highway.

To avoid the appearance of inconsistent distribution, the horizontal alignment should be coordinated carefully with the profile design.

30.7.2 Superelevation

Superelevation is predicated on design speed and all highways shall be superelevated according to their speeds rather than using a superelevation for a single radius for all design speeds.

A 6 percent maximum superelevation rate shall be used on urban freeways. A 4 percent maximum superelevation rate may be used on high-speed (greater than 40 Mph) urban highways to minimize conflicts with adjacent development and intersecting streets. Low speed (40 Mph or less) urban streets can use a 4 percent or 6 percent maximum superelevation rate.

Values for superelevation for urban freeways shall be in accordance with Table 30-F.

Table 30-F:
Values of Superelevation for Urban Freeways

RADIUS (FT.)	SUPERELEVATION (PERCENT) FOR DESIGN SPEEDS OF							
	30 MPH	35 MPH	40 MPH	45 MPH	50 MPH	55 MPH	60 MPH	70 MPH
275	6.0							
300	6.0							
400	5.6	6.0						
500	5.1	5.7						
600	4.7	5.4	5.9					
700	4.4	5.1	5.7	6.0				
800	4.1	4.8	5.4	5.9				
900	3.9	4.5	5.1	5.7	6.0			
1000	3.7	4.3	4.9	5.5	5.9			
1200	3.3	3.9	4.5	5.0	5.5	5.9		
1400	2.9	3.6	4.1	4.7	5.2	5.7	6.0	
1600	2.7	3.3	3.8	4.4	4.9	5.4	5.9	
1800	2.4	3.0	3.6	4.1	4.6	5.1	5.6	
2000	2.2	2.8	3.3	3.8	4.3	4.9	5.4	6.0
2500	1.8	2.3	2.8	3.3	3.8	4.3	4.8	5.8
3000	1.6	2.0	2.4	3.0	3.4	3.9	4.3	5.3
3500	1.5	1.8	2.1	2.6	3.0	3.5	4.0	4.9
4000	NC	1.5	1.9	2.3	2.7	3.1	3.6	4.4
4500		NC	1.7	2.1	2.5	2.9	3.3	4.1
5000			1.6	1.9	2.2	2.6	3.0	3.7
6000			NC	1.6	1.9	2.2	2.6	3.2
7000				NC	1.7	2.0	2.3	2.8
8000					1.5	1.7	2.0	2.5
9000					NC	1.6	1.8	2.3
10000						NC	1.7	2.1
12000							NC	1.7
14000								1.5

16000	NC
18000	
19000	

NC=Normal Crown

No Superelevation Required When Radius (Ft.) is Greater Than:

	30 MPH	35 MPH	40 MPH	45 MPH	50 MPH	55 MPH	60 MPH	70 MPH
6% Max.	3153	4133	5247	6497	7883	9423	11111	14046

NOTE: Superelevation Rates Less Than 1.5 percent Shall Not Be Used
Values for superelevation for urban highways shall be in accordance with Table 30-G.

Table 30-G:
Values of Superelevation for Urban Highways

SUPERELEVATION (PERCENT) FOR DESIGN SPEEDS OF										
	30 MPH		35 M.P.H		40 MPH		45 MPH	50 MPH	55 MPH	60 MPH
RAD. (FT.)	4% max	6% max	4% max	6% max	4% max	6% max	4% max	4% max	4% max	4% max
215		6.0								
250		2.0								
275		NC								
300	4.0									
325	4.0			5.6						
350	3.9			3.8						
400	3.8			NC						
450	3.7		4.0			6.0				
500	3.6		3.9			3.6				
600	3.4		3.8		4.0	NC				
700	3.2		3.6		3.9					
800	3.0		3.4		3.8		4.0			
900	2.9		3.2		3.6		3.9			
1000	2.7		3.1		3.5		3.8	4.0		
1200	2.5		2.9		3.2		3.6	3.9	4.0	
1400	2.4		2.7		3.0		3.4	3.7	3.9	
1600	2.2		2.6		2.9		3.2	3.5	3.8	
1800	2.1		2.4		2.7		3.0	3.3	3.7	4.0
2000	2.0		2.3		2.6		2.9	3.2	3.5	3.8
2400	1.7		2.1		2.4		2.7	2.9	3.3	3.6

2800	1.5	1.9	2.2	2.5	2.7	3.0	3.4
3200	NC	1.7	2.0	2.3	2.6	2.9	3.2
3600		1.6	1.9	2.2	2.4	2.7	3.0
4000		NC	1.7	2.0	2.3	2.6	2.8
4500			1.6	1.9	2.1	2.4	2.7
5000			NC	1.7	2.0	2.3	2.5
6000				1.5	1.7	2.0	2.3
7000				NC	1.5	1.8	2.0
8000					NC	1.6	1.8
9000						1.5	1.7
10000						NC	1.5
12000							NC
14000							

The minimum superelevation to be used is 1.5 percent on flat radius curves requiring superelevation ranging from 1.5 percent to 2 percent. The superelevation should be increased by 0.5 percent in each successive pair of lanes on the low side of the superelevation when more than two lanes are superelevated in the same direction. Superelevation shall not normally be used on local or other roadway classifications with a design speed of 40 Mph or less.

30.7.2.1 Maximum Curvature for Normal Crown Road

Table 30-H is referenced from **AASHTO Table III-13**, and provides Maximum Curvature for Normal Crown Section:

Table 30-H:
Maximum Curvature for Normal Crown Section

				SIDE FRICTION FACTOR, F, WITH ADVERSE CROWN	
Design Speed (MPH)	Avg. Running Speed (MPH)	Max. Curve Degrees	Min. Curve Radius (ft)	At Design Speed	At Running Speed
20	20	3^{o}23’	1,700	.031	.031
30	28	1^{o}43’	3,340	.033	.031
40	36	1^{o}02’	5,550	.034	.031
50	44	0^{o}41’	8,320	.035	.031
55	48	0^{o}35’	9,930	.035	.031
60	52	0^{o}29’	11,690	.035	.030
65	55	0^{o}26’	13,140	.035	.030
70	58	0^{o}23’	14,690	.037	.030

30.7.2.2 Superelevation Transition

The superelevation transition generally consists of the superelevation runoff (length of roadway needed to accomplish the change in cross slope from a normal crown section to a fully superelevated section or vice versa). Defining or establishing superelevation runoff shall be in accordance with **AASHTO Green Book.**

30.7.3 Curvature

The changes in direction along a highway are accounted for by simple curves or compound curves. Excessive curvature or poor combinations of curvature generates accidents, limits capacity, causes economic losses in time and operating costs, and detracts from a pleasing appearance. Broken-back curves should be avoided.
Street curvature shall meet the minimum specifications shown in Table 30-I.

Table 30-I:
Minimum Horizontal Street Curve Specifications

DESIGN CRITERIA	LOCAL STREET	COLLECTOR STREET	ARTERIAL STREET
Minimum Design Speed	20 mph	35 mph	40 mph
Minimum Centerline Radius	100 ft	300 ft	500 ft
Minimum Reverse Curve Tangent	50 ft	100 ft	200 ft
Minimum Intersection Approach Tangent	100 ft	200 ft	300 ft

For additional information and criteria relative to horizontal alignment, refer to the **AASHTO Green Book**, current edition.

30.8 Vertical Alignment

30.8.1 General

The profile line is a reference line by which the elevation of the pavement and other features of the highway are established. It is controlled mainly by topography, type of highway, horizontal alignment, safety, sight distance, construction costs, cultural development, drainage, and pleasing appearance. The performance of heavy vehicles on a grade must also be considered. All portions of the profile line must meet sight distance requirements for the design speed of the road.

In flat terrain, the elevation of the profile line is often controlled by drainage considerations. In rolling terrain, some undulation in the profile line is often advantageous, both from the standpoint of truck operation and construction economy. This should be done with appearance in mind (i.e., a profile on tangent alignment exhibiting a series of humps visible for some distance ahead should be avoided whenever possible). In rolling terrain, however, the profile usually is closely dependent upon physical controls. In considering alternative profiles, economic comparisons should be made.

30.8.2 Position with Respect to Cross Section

The profile line should generally coincide with the axis of rotation for superelevation. Its relation to the cross section should be as follows:

- Undivided Highways -The profile line should coincide with the highway centerline.
- Ramps and Freeway-to-Freeway Connections - The profile line may be positioned at either edge of pavement, or centerline of ramp if multi-lane.
- Divided Highways - The profile line may be positioned at either the centerline of the median or at the median edge of pavement. The former case is appropriate for paved medians 30 ft. wide or less. The latter case is appropriate when:
  - The median edges of pavement of the two roadways are at equal elevation.
  - The two roadways are at different elevations.

30.8.3 Permissible Roadway Grades

The minimum allowable grade for roadways or alleys is 0.5 percent. The minimum allowable grade for bubbles or cul-de-sacs within the bulb is 1 percent. The maximum allowable grade for any roadway is shown in Table 30-J.

Table 30-J:

Maximum Allowable Grades

DESCRIP-TION	DESIGN SPEED	MAXI-MUM GRADE	K VALUE CREST	RANGES SAG	MIN. CREST	V.C.L. SAG
Local	35	8	35-50	40-50	50	50
Minor Collector	35	7	35-50	40-50	50	50
Major Collector	40	7	55-65	55-65	50	50
Minor Arterial	45	6	70-105	65-85	70	60
Principal Arterial	55	6	115-220	90-125	110	90
Freeway	60	5	160-300	105-155	150	100

Note: The maximum grade may be modified on a case by case basis in areas where steep hills and grades are the norm and the indicated rates may be impossible to attain.

30.8.4 Permissible Intersection Grades

The maximum permissible intersection approach grade (min. 50 ft.) should be 4 percent. For signalized intersection approaches, grades should not exceed 2 percent within 50 ft. of intersection. Exceptions will be on a case by case basis.

30.8.5 Vertical Curves

Properly designed vertical curves should provide adequate sight distance, safety, comfortable driving, good drainage, and pleasing appearance. Vertical curves shall be designed in accordance with the **AASHTO Green Book**.

Vertical curves are not required where an algebraic difference in grade is less than 0.35 percent. Vertical curves that have a level point and flat sections near their crest or sag should be evaluated for drainage. Values of K=167 or greater should be checked for drainage. All vertical curves shall be labeled in the profile with the station of the vertical point of intersection (VPIS), the elevation (PVIE), the length of vertical curve (VCL), K=(L/A) and the middle ordinate (m).

30.9 Combination of Horizontal and Vertical Alignment

To avoid the possibility of introducing serious hazards, coordination is required between horizontal and vertical alignment. Particular care must be exercised to maintain proper sight distance. Where grade line and horizontal alignment will permit, it is desirable to superimpose vertical curves on horizontal curves. This reduces the number of sight distance restrictions and makes changes in the profile less apparent (particularly in rolling terrain). Care should be taken, however, not to introduce a sharp horizontal curve near a pronounced crest or grade sag (this is particularly hazardous at night).

In cases where curves sharper than 7 degrees are located on steep grades, it is considered good design to flatten the grade slightly throughout the length of the curve. Horizontal curvature and profile grade should be made as flat as possible at highway intersections.

On divided highways, variation in the width of medians, the use of separate profiles, and horizontal alignment should be considered to achieve the design and operational advantages of one-way roadways.

NOTE: Changes in noise level should be evaluated where noise receptors are present.

30.10 Lane Transition

Design standards of the various features of the transition between roadways of different widths should be consistent with the design standards of the superior roadway. The transition should be made on a tangent section whenever possible and should avoid locations with horizontal and vertical sight distance restrictions. Whenever feasible, the entire transition should be visible to the driver of a vehicle approaching the narrower section. The design should be such that at-grade intersections within the transition are avoided.

The information below shows the minimum required taper length based upon the design speed of the roadway. In all cases, a taper length longer than the minimum should be provided where possible. When tapering the transition drops, a lane should be on the right so that traffic merges to the left.

For design speed greater than 40 MPH:

$L = V\,W$

For design speed equal to or less than 40 MPH:

$L = V \times V \times W / 60$

Where V = Design Speed (MPH)
W = Lane Width Reduction (FT)
L = Taper Length (FT)

30.11 Major Cross Section Elements

30.11.1 General

The major cross section elements considered in the design of streets and highways include the pavement surface type, cross slope, lane widths, shoulders, roadside or border, curbs, sidewalks, driveways, and medians. NOTE: For additional information and criteria relative to major cross section items, refer to the **AASHTO Green Book**.

30.11.2 Standard Roadway Elements Width:

Minimum requirements are listed in Table 30-K below for new street construction, however every effort should be made to upgrade the existing streets to bring them to the current Department standard as much as practical.

Table 30-K:
Standard Roadway Elements Widths

ROADWAY	THE STANDARD WIDTH BASED ON DDOT GUIDELINE
The Minimum ROW for One-way travel Road	55’ with 10’ setback both sides
The Minimum ROW for Two-way Travel Road	75’ with 10’ setback both sides
Two-way Street, one lane each, with Parking both sides	36’ Paved Surface Width (Prefer 38’)
Two-way Street one lane each with one side parking	32’ Paved Surface Width (Prefer 34’)
One-way Street one lane with two side parking	30’ Paved Surface Width
One-way Street one lane with one side parking	22’ Paved Surface Width
Driving Lane	10’ to 12’ Paved Surface Width
Driving Lane having Buses	11’ Paved Surface Width
Driving Lane, with parking	18’ Paved Surface Width*
Driving Lane, with Parking and Have Buses	19’ Paved Surface Width
Parking Lane	8’ Paved Surface Width
Bicycle Lane one way	5’ Paved Surface Width

Bicycle Lane two ways	8’ Paved Surface Width
Shared Use Path (Two-Way)	10’ - 12’ Paved Surface Width (14’ if heavy use)
Sidewalk Pavement	6’ Paved Surface Width**
Sidewalk including 4-foot tree space	10’ Surface Width**
Middle of Road median	4’ Minimum Surface Width
Shoulder Width	10’ Surface Width

*Driving and parking lane width together = 19 ft. (11 ft. adjacent maneuvering lane). 8 ft. is the standard width for parallel parking and requires a minimum 11 ft. adjacent maneuvering lane, without going over the adjacent travel lane and/or double yellow lines. When the parallel parking lane is narrower than the standard 8 ft. wide parallel lane, than it requires another foot added to the adjacent maneuvering lane. For example, if the parallel parking lane is 7 ft. wide, than the adjacent maneuvering lane must be 12 ft. wide.

**Each street in the District has a designated sidewalk width for public space. The minimum designated width is 10 ft., which includes a 6 ft. sidewalk and 4 ft. tree space. However, each street within the District of Columbia has a designated sidewalk width for each street, and if a particular street has a higher designated minimum width than 10 ft. minimum width then higher width will become the minimum width for that street and it must be met. If there is no designated sidewalk width, then the minimum 10 ft. designated sidewalk width must be used. (Note: The sidewalk width usually includes the tree space).

30.13 Lane Widths

Lane widths have a great influence on driving safety and comfort. On freeways the predominant lane width is 12 ft. Although lane widths of 12 ft. are desirable, there are circumstances that necessitate the use of lanes less than 12 ft. on city streets. In urban areas, the use of 11 ft. wide lanes is acceptable. 10 ft. wide lanes have been provided in the past at certain locations where ROW and existing development became stringent controls and where truck volumes were limited. However, 10 ft. wide lanes would not be proposed today for new street construction. 10 ft. lanes may also be used adjacent to bicycle lanes if bus and truck traffic is not substantial. To help accommodate a bicycle, the outside (curb) lane should be wider than the inside lane(s), 14 ft. where possible. For example, a 50 ft. wide street with 4 lanes of traffic should have 14 ft. outside lanes and 12 ft. inside lanes.

Where alternate bike access is provided, the outside lane width should be 1 ft. wider than the adjacent thru-lane width. When rehabilitating or reconstructing existing highways with lane widths of 10 ft. or less, the existing lanes should be widened to either an 11 ft. lane, minimum, or 12 ft. lane, that is desirable. Currently, there are only two minor arterial streets that have 9 ft. wide lanes. They are: E Street, NW, between 5^{th} Street and 13^{th} Street and H Street, NW between 5^{th} and 13^{th} Streets.

30.14 Roadway-Rail Grade Crossings

All roadway/rail crossings should be coordinated with the railroad to provide a consistent surface and traffic control. To properly accommodate bicyclists, at grade roadway/rail crossings should be at right angle to the rails. If the crossing is less than 45 degrees, an additional paved shoulder should be provided to permit the bicyclist to cross the track at a safer angle. Refer to the **AASHTO Guide for the Development of Bicycle Facilities** for additional information.

The railroad company will provide the design and special provisions for inclusion in the contract plans. The District will provide funds for construction of the Roadway-Rail Grade Crossing with participation of FHWA.